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DESIGN CURVES FOR ANCHORED STEEL SHEET PILING

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WATERWAYS DIVISION

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

DESIGN CURVES FOR ANCHORED STEEL
SHEET PILING

BY WALTER C. BOYER,¹ J. M. ASCE, AND
HENRY M. LUMMIS, III,² J. M. ASCE

SYNOPSIS

The design of anchored steel sheet piling bulkheads is a procedure normally encountered in many pier development projects. Although no design method has met with universal acceptance, it is felt that the "free support method" will yield a sufficiently rational approach for preliminary design considerations. A set of curves introduced with this paper will simplify preliminary design calculations.

INTRODUCTION

Anchored sheet piling depends for its stability on the embedment of the lower part of the sheeting assisted by an anchorage system as near to the top as is consistent with the durability of the anchor rods. Such reactions, therefore, are yielding supports under the effects of the backfill and surcharge forces. The sheeting must be of sufficient strength to sustain the lateral pressures involved. Since walls of this type experience considerable yield, it is rational to assume that the soil attains Rankine's active state.³ This assumption forms the basis for the normal methods of design as demonstrated by Raymond P. Pennoyer⁴ and others.

A set of curves provides an excellent basis for the preliminary design of sheet pile bulkheads. Consistent with certain assumptions, it is possible to develop such curves quite readily for the major factors involved in sheet pile design, provided the relationships involved are developed in efficient form for calculation.

NOTE.—Written comments are invited for publication; the last discussion should be submitted by July 1, 1953.

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³ "Fundamentals of Soil Mechanics," by Donald W. Taylor, John Wiley & Sons, Inc., New York, N. Y., 1948, p. 487.

⁴ "Design of Steel Sheet-Piling Bulkheads," by Raymond P. Pennoyer, *Civil Engineering*, Vol. 3, 1933, pp. 615-619.

SELECTION OF DESIGN PROCEDURE

The three primary factors of interest in anchored sheet pile design are the depth of embedment, the bending moment in the sheeting, and the anchor pull. There is varied thought as to the procedure to be followed in the determination of these factors. The "free earth support method" produces minimum penetration depth resulting in increased bending moment. The "fixed earth support method" produces increased penetration depth, resulting in constraining action of the earth with resultant reduction in bending moment. The material utilization factor is essentially the same in either case but the latter method is purported to give a greater factor of safety against toe "push out," and reduced cost of the anchorage system.

In the development of passive resistance it has become quite popular to increase the Rankine passive coefficient by a factor of two as recommended by H. Blum based on the tests of O. Franzius.⁵ This practice has been questioned by Gregory P. Tschebotarioff,⁶ M. ASCE, who terms it "a dangerous oversimplification." The writers concur completely in this statement.

On the basis of the foregoing considerations, the method of design utilized in this paper is the "free earth support method" in which the penetration is obtained by taking moments about the anchor point but without doubling the increment of passive pressure. Under this condition, the driving costs are reduced while the depth of penetration is still adequate. Moreover, experience indicates that the additional cost of anchorage is money well spent. In several sheet pile installations that exhibited signs of distress, it was observed that the anchorage system had proved to be inadequate for the surcharge forces involved.

BASIC ASSUMPTIONS

Fig. 1 indicates the cross section of a typical sheet pile wall with its anchorage system. The variables involved in the design are as follows: H_s is the height of surcharge; H is the height of sheet pile above water; H_w is the depth of water; ϕ is the angle of internal friction of dry soil; ϕ' is the angle of internal friction of wet soil; W is the weight of dry soil; W' is the buoyed-up weight of soil; H_p is the depth of penetration; T is the tension in tie-rod pounds per foot of wall; and M is the maximum bending moment per foot of wall.

The symbols H_s , H , and H_w , as well as the factors defining the soil properties, denote situation variables obtained prior to actual design. The first problem is to solve for the depth of penetration, H_p , necessary to hold the wall in equilibrium; secondly, the tension, T , in the tie rod must be determined; and, finally, the maximum bending moment, M , must be found. Other design considerations such as the anchorage system, wales, and incidental details are not a part of this paper.

In the development of the formulas that follow, the following assumptions are made to simplify the problem:

⁵ "Versuche mit Passiven Druck," by O. Franzius, *Der Bauingenieur*, 1924.

⁶ "Final Report—Large Scale Earth Pressure Tests with Model Flexible Bulkheads," by Gregory P. Tschebotarioff, Princeton, N. J., January 31, 1949, p. 61.

1. The specific gravity of the soil particles is assumed to be equal to 2.65—a commonly accepted constant.

2. The placement of the tie is assumed at the mean water level. The tie rod is normally placed at or near the mean water level in the zone of saturation to reduce corrosion effects. It is undesirable to place the tie deeper than 2 ft below water level since workers must be paid divers' wages when working below this depth.⁷

3. The Rankine coefficients are assumed to be applicable. Since sheet pile installations have yielding supports, it is rational to assume that they deflect sufficiently to develop the active and passive Rankine coefficients.

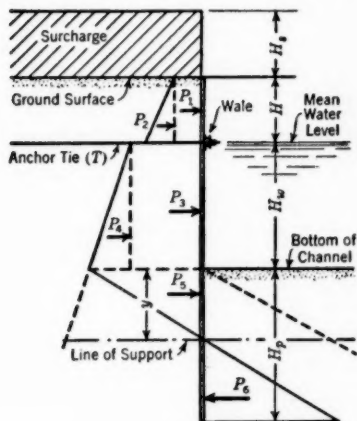


FIG. 1

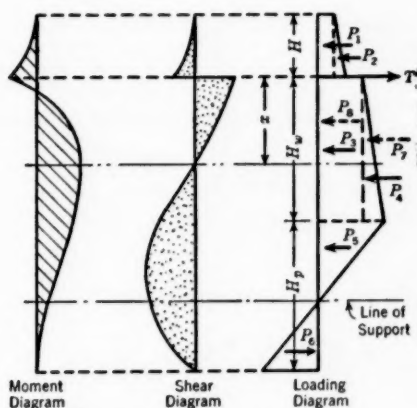


FIG. 2

4. The soil is considered to have a reduced friction angle for the submerged condition, an assumption which is consistent with usual design procedure. Paul Andersen,⁸ M. ASCE, states:

"This design conventionality may be justified by the fact that the upper portion of the backfill is usually made up of selected material, while the lower portion is the natural soil considerably disturbed by the construction."

5. The soil is assumed to be homogeneous. This assumption is necessary in the development of the curves and is fundamental to most design procedures. However, if boring data indicate definite soil stratification, the curves developed are not applicable.

DEVELOPMENT OF RELATIONSHIPS

With due regard to these five assumptions, the development of the relationships upon which the curves are based, follows: The Y -distance to the

⁷ *U. S. Steel Sheet Piling*, Carnegie-Illinois Steel Corp., p. 9.

⁸ "Substructure Analysis and Design," by Paul Andersen, Irwin-Farnham Pub. Co., Chicago, Ill., 1948, p. 49.

line of support (Fig. 1) is readily obtained by setting the active pressure equal to the passive pressure; thus,

$$Y W' K_3 = H_s W K_2 + H W K_2 + H_w W' K_2 + Y W' K_2 \dots (1)$$

Solving Eq. 1,

$$Y = \frac{K_2 (H_s W + H W + H_w W')}{W' (K_3 - K_2)} \dots (2)$$

in which

$$\left. \begin{aligned} K_1 &= \frac{1 - \sin \phi}{1 + \sin \phi} \\ K_2 &= \frac{1 - \sin \phi'}{1 + \sin \phi'} \\ K_3 &= \frac{1 + \sin \phi'}{1 - \sin \phi'} \end{aligned} \right\} \dots (3)$$

The various forces exerted on the wall are evaluated in the following manner:

$$\left. \begin{aligned} P_1 &= W H H_s K_1 \\ P_2 &= \frac{W H^2 K_1}{2} \\ P_3 &= W H_w K_2 (H_s + H) \\ P_4 &= \frac{W' H_w^2 K_2}{2} \\ P_5 &= \frac{K_2 (H_s W + H W + H_w W')^2}{2 W' (K_3 - K_2)} \\ P_6 &= [W' H_p (K_3 - K_2) - K_2 (H_s W + H W + H_w W')] \left[\frac{H_p}{2} - \frac{Y}{2} \right] \end{aligned} \right\} (4)$$

The solution for H_p is obtained by taking moments about the tie; thus,

$$\begin{aligned} \sum M (\text{tie}) &= P_1 \frac{H}{2} + P_2 \frac{H}{3} - P_3 \frac{H_w}{2} - P_4 \frac{2 H_w}{3} - P_5 \left(H_w - \frac{Y}{3} \right) \\ &+ P_6 \left(H_w + H_p - \frac{H_p}{3} + \frac{Y}{3} \right) = 0 \dots (5) \end{aligned}$$

Substituting for the P -forces the expressions in Eqs. 4, and simplifying, Eq. 5 becomes

$$\begin{aligned} &H_p^3 \frac{W' (K_3 - K_2)}{3} + H_p^2 \frac{H_w W' (K_3 - K_2)}{2} - \frac{H_p^2 K_2 (H_s W)}{2} \\ &- \frac{H_p^2 K_2 (H W + H_w W')}{2} - H_p H_w K_2 (H_s W + H W + H_w W') \\ &+ \frac{W H^2 H_s K_1}{2} + \frac{W H^3 K_1}{6} - \frac{H_w^2 K_2 (3 H_s W + 3 H W + 2 H_w W')}{6} = 0 \dots (6) \end{aligned}$$

Eq. 6 can be expressed in the general form:

$$f(H_p, H, H_s, H_w, \phi, \phi', W, W') = 0 \quad (7)$$

By applying the principles of dimensional analysis, Eq. 7 can be reduced to an equation of dimensionless variables, thereby reducing the number of variables by the number of primary quantities involved.⁹ Since there are two primary quantities in Eq. 6—force and length—an equation can be obtained, involving six dimensionless variables, as follows:

$$f' \left(\frac{H_p}{H_w}, \frac{H}{H_w}, \frac{H_s}{H_w}, \phi, N, \frac{W}{W'} \right) = 0 \quad (8)$$

in which $N = \phi'/\phi$. However, observing that W is the product of a constant and the specific gravity of the soil particles (G), and that W' is the product of the same constant and the quantity $G - 1$ —

$$\frac{W}{W'} = \frac{G}{G - 1} \quad (9)$$

Since the specific gravity has been assumed constant, this ratio is also constant. Therefore, if both ϕ and N are considered constant, the general formula, Eq. 9, reduces to

$$f'' \left(\frac{H_p}{H_w}, \frac{H}{H_w}, \frac{H_s}{H_w} \right) = 0 \quad (10)$$

Calling $\frac{H_p}{H_w} = C_p$; $\frac{H}{H_w} = C$; $\frac{H_s}{H_w} = C_s$; $A = (K_3 - K_2)$; $B = \frac{K_2 W}{2 W'}$; and $D = \frac{K_1 W}{2 W'}$, and dividing Eq. 6 by $W' H_w^3$, the following expression is obtained:

$$C_p^3 \frac{A}{3} + C_p^2 \frac{A}{2} - C_p^2 \left(C_s B + C B + \frac{K_2}{2} \right) - C_p (2 C_s B + 2 C B + K_2) + C^2 C_s D + C^3 \frac{D}{3} - C_s B - C B - \frac{K_2}{3} = 0 \quad (11)$$

Simplifying Eq. 11 and arranging it in the most convenient form:

$$C_s = \frac{\left[C_p^3 \frac{A}{3} + C_p^2 \left(\frac{A - K_2}{2} \right) - C_p K_2 - \frac{K_2}{3} \right]}{B (C_p^2 + 2 C_p + 1) - C^2 D} - \frac{C [B (C_p^2 + 2 C_p + 1)] + C^3 \frac{D}{3}}{B (C_p^2 + 2 C_p + 1) - C^2 D} \quad (12)$$

The next unknown to be determined is the tension in the tie. If an equation of equilibrium is written for horizontal forces (Fig. 1),

$$T = P_1 + P_2 + P_3 + P_4 + P_5 - P_6 \quad (13)$$

⁹ "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, McGraw-Hill Book Co., Inc., New York, N. Y., 1938, pp. 14-24.

Substituting the values for the P -forces (Eq. 4), and simplifying,

$$T = W H H_s K_1 + \frac{W H^2 K_1}{2} + W H_w H_s K_2 + W H_w H K_2 + W' \frac{H_w^2 K_2}{2} - \frac{W' H_p^2 (K_3 - K_2)}{2} + K_2 H_p (H_s W + H W + H_w W') \dots (14)$$

or, in general form:

$$f(T, H_s, H, H_w, H_p, \phi, N, W, W') = 0 \dots (15)$$

Taking advantage of the principles of dimensional analysis once again, and considering ϕ and N constant,

$$f' \left(\frac{T}{W H_w^2}, \frac{H}{H_w}, \frac{H_s}{H_w}, \frac{H_p}{H_w} \right) = 0 \dots (16)$$

Therefore, Eq. 14 is divided by $W H_w^2$, calling

$$C_t = \frac{T}{W H_w^2} \dots (17a)$$

$$E = \frac{W' (K_3 - K_2)}{2 W} \dots (17b)$$

and

$$F = \frac{K_2 W'}{2 W} \dots (17c)$$

Thus,

$$C_t = 2 F C_p + F - C_p^2 E + K_2 (C_p + 1)(C_s + C) + C K_1 \left(C_s + \frac{C}{2} \right) \dots (18)$$

The final value that must be computed is the maximum bending moment in the pile. Formulas must be developed for the bending moment at three different points—at the tie, between the tie and the bottom of the channel, and between the bottom of the channel and the bottom of the pile. Fig. 2 shows the loading on the piling and the resulting shear and moment diagram.

Assuming that the maximum moment occurs at a point, distance x below the tie (in which x is less than H_w), the first step is to locate this point by determining the section where the summation of shear forces is equal to zero. That is,

$$P_1 + P_2 + P_7 + P_8 - T = 0 \dots (19)$$

in which

$$\left. \begin{aligned} P_7 &= \frac{W x^2 K_2}{2} \\ P_8 &= W x K_2 (H_s + H) \end{aligned} \right\} \dots (20)$$

and

Substituting the values of the P -forces in Eq. 20, Eq. 19 becomes

$$W H H_s K_1 + \frac{W H^2 K_1}{2} + \frac{W' x^2 K_2}{2} + W x K_2 (H_s + H) - T = 0 \dots (21)$$

Taking advantage of the fact again that W/W' is constant, and considering ϕ and N constant, Eq. 21 is divided by $W H^2 w$ which reduces the number of variables by two; thus,

$$C_x^2 F + C_x [K_2 (C_s + C)] + \left[C C_s K_1 + C^2 \frac{K_1}{2} - C_t \right] = 0 \dots (22)$$

in which $C_x = \frac{x}{H_w}$.

Solving for C_x ,

$$C_x = \frac{-K_2 (C_s + C) \pm \sqrt{[K_2 (C_s + C)]^2 - 4 F \left(C C_s K_1 + C^2 \frac{K_1}{2} - C_t \right)}}{2 F} \dots (23)$$

Taking moments about the point thus obtained,

$$\begin{aligned} M &= P_1 \left(\frac{H}{2} + x \right) + P_2 \left(\frac{H}{3} + x \right) - T x + P_3 \frac{x}{2} + P_7 \frac{x}{3} \\ &= \frac{W H^2 H_s K_1}{2} + W H H_s x K_1 + \frac{W H^3 K_1}{6} + \frac{W H^2 x K_1}{2} - T x \\ &\quad + \frac{W x^2 K_2 H_s}{2} + \frac{W x^2 K_2 H}{2} + \frac{W' x^3 K_2}{6} \dots (24) \end{aligned}$$

Once again dimensional analysis is employed, and since ϕ and N are considered constant, Eq. 24 is divided by $W H^3 w$ and arranged in the most convenient form:

$$C_m = C^2 C_s \frac{K_1}{2} + C^3 \frac{K_1}{6} - C_x \frac{2}{3} F - C_x^2 \left[\frac{K_2}{2} (C_s + C) \right] \dots (25)$$

in which $C_m = \frac{M}{W H^3 w}$.

By taking moments about the tie and reducing to dimensionless form, the equation for moment at the tie is

$$C_m = C^2 C_x \frac{K_1}{2} + C^3 \frac{K_1}{6} \dots (26)$$

Eq. 26 is the same as the first two terms of Eq. 25.

If the point of zero shear falls below the bottom of the channel, Eq. 25 is not valid and another equation must be derived. Fig. 3 shows that part of the piling below the channel with the loads acting on it. Since P_9 must equal P_8 ,

$$U = 2 (H_p - Y) = 2 H_p - \frac{2 K_2}{W' (K_3 - K_2)} (H_s W + H W + H_w W') \dots (27)$$

Applying dimensional analysis to Eq. 27, divide through by H_w and simplify,

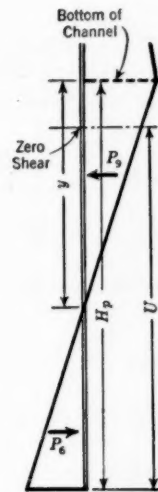


FIG. 3

Let $U/H_w = C_u$ and

$$C_u = 2 C_p - \frac{K_2}{E} (C_s + C) - \frac{2 K_2}{A} \dots (28)$$

Taking moments about the point of zero shear,

$$M = \frac{5 P_2}{3} (H_p - y) - \frac{P_2}{3} (H_p - y) \dots (29)$$

and

$$M = \frac{U^2}{24} \left[5 W' H_p (K_3 - K_2) - 5 K_2 (W H_s + W H + W' H_w) - W' (K_3 - K_2) \frac{U}{2} \right] \dots (30)$$

Consider ϕ and N constant and divide Eq. 30 by $W H_s^3$ to convert Eq. 30 into dimensionless terms:

$$C_m = \frac{C_u^2}{4.8} \left[2 E C_p - K_2 (C_s + C) - 2 F - \frac{E C_u}{5} \right] \dots (31)$$

Simplifying,

$$C_m = \frac{E}{6} C_u^3 \dots (32)$$

In reviewing the relationships developed, it is well to note that the dependent variables are all functions of the same independent variables when ϕ and ϕ' are considered constant. Note that in Eq. 12

$$C_p = f(C, C_s) \dots (33a)$$

in Eq. 18,

$$C_t = f(C, C_s, C_p) = f'(C, C_s) \dots (33b)$$

and, in Eq. 25,

$$C_m = f(C_s, C, C_s) \dots (33c)$$

—but, since (in Eq. 23),

$$C_x = f(C, C_s, C_t) = f'(C, C_s) \dots (34)$$

then

$$C_m = f(C, C_s) \dots (35)$$

Likewise, in Eq. 26,

$$C_m = f(C, C_s) \dots (36a)$$

and, in Eq. 32,

$$C_m = f(C_u) \dots (36b)$$

—but, since

$$C_u = f(C_p, C, C_s) = f'(C, C_s) \dots (37)$$

then

$$C_m = f(C, C_s) \dots (38)$$

Since it has been demonstrated that C_p , C_t , and C_m are functions of C and C_s alone, when ϕ and ϕ' are considered constant, lines of constant C_p , C_t ,

and C_m can be plotted with C the ordinate and C_s the abscissa. This procedure lends itself to a very efficient construction of diagrams.

METHOD OF DEVELOPING DATA FOR CHARTS

The curves were plotted with values of C_s as the abscissas and values of C as the ordinates, each set for a particular value of ϕ and N ($N = \phi'/\phi$). Each set provided the constants, C_p , C_t , and C_m , over a practical range of values, by which the depth of penetration, the tension force in the tie rod per foot, and the maximum bending moment per foot may be determined for a particular problem.

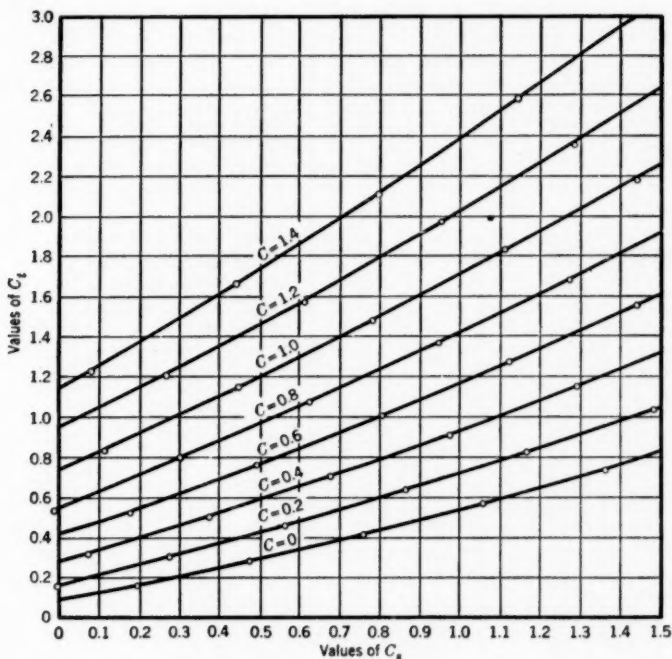


FIG. 4.—SAMPLE INTERPOLATION CURVE FOR C_t ($\phi = 30^\circ$, $N = \frac{1}{2}$)

The first series of solutions was made for C_p utilizing Eq. 12. Actually, by assuming constant values for C_p and varying C_t values for C_s were determined. By considering C_s as the dependent variable, the necessity of solving a cubic equation was avoided.

The next parameter considered was C_t . Using the values of C , C_s , and C_p previously determined, Eq. 18 yielded odd values of C_t . Therefore, a series of interpolation curves was drawn with values of C_t as the ordinates and values of C_s as the abscissas (Fig. 4). Thus, for any desired value of C_t , C_s was determined from the interpolation curves with C as the parameter.

A series of solutions for C_m was found by using Eqs. 25, 26, or 32, after having determined the location of the maximum moment by Eqs. 23 and 28. A set of interpolation curves was drawn for C_m similar to those for C_t in Fig. 4.

A complete log of the data for the condition of $\phi = 30^\circ$ and $N = \frac{3}{4}$ is presented in Table 1. Similar data^{9a} were determined for three values of ϕ

TABLE 1.—ANCHORED STEEL SHEET PILING DATA FOR CONSTRUCTION
DESIGN CURVES ($\phi = 30^\circ$ AND $N = \frac{3}{4}$)

$C = H/H_w$	$C_s = H_s/H_w$ (Eq. 17a) (1)	$C_t = z/H_w$ (Eq. 17a) (2)	$C_x = x/H_w$ (3)	$C_u = U/H_w$ (Eq. 32) (4)	C_m (Eq. 32) (5)	$C_s = H_s/H_w$ (Eq. 17a) (1)	$C_t = z/H_w$ (Eq. 17a) (2)	$C_x = x/H_w$ (3)	$C_u = U/H_w$ (Eq. 32) (4)	C_m (Eq. 32) (5)
(a) $C_p = H_p/H_w = 0.8$						(b) $C_p = H_p/H_w = 1.0$				
0	0.195	0.1636	0.8151	0.0793	0.4710	0.2802	0.8518	0.1338
0.2	-0.005	0.1670	0.8058	0.0765	0.2735	0.3074	0.8536	0.1327
0.4	0.0771	0.3220	0.8575	0.1314
(c) $C_p = H_p/H_w = 1.2$						(d) $C_p = H_p/H_w = 1.4$				
0	0.7580	0.4143	0.8941	0.2018	1.0580	0.5693	0.9426	0.2878
0.2	0.5624	0.4631	0.8981	0.2005	0.8650	0.6424	0.9483	0.2870
0.4	0.3705	0.5035	0.9092	0.1986	0.6770	0.7093	0.9627	0.2843
0.6	0.1790	0.5324	0.9174	0.1954	0.4922	0.7613	0.9677	0.2711
0.8	-0.0174	0.5407	0.9185	0.1892	0.3038	0.8056	0.9810	0.2642
1.0	0.1186	0.8403	0.9935	0.2564
1.2	-0.0804	0.8431	1.4060	0.2500
(e) $C_p = H_p/H_w = 1.6$						(f) $C_p = H_p/H_w = 1.8$				
0	1.3640	0.7398	0.9899	0.3887
0.2	1.1680	0.8285	0.9928	0.3840	1.4810	1.0380	1.7569	0.5049
0.4	0.9780	0.9134	1.5993	0.3808	1.2970	1.1540	1.7441	0.4940
0.6	0.8040	1.0090	1.5785	0.3662	1.1220	1.2750	1.7241	0.4771
0.8	0.6270	1.0829	1.5601	0.3535	0.9490	1.3780	1.7025	0.4594
1.0	0.4500	1.1550	1.5417	0.3411	0.7810	1.4830	1.6769	0.4390
1.2	0.2700	1.2100	1.5257	0.3305	0.6130	1.5770	1.6513	0.4192
1.4	0.0824	1.2390	1.5158	0.3242	0.4440	1.6700	1.6265	0.4006
(g) $C_p = H_p/H_w = 2.0$						(h) $C_p = H_p/H_w = 2.2$				
0.6	1.4440	1.5533	1.8665	0.6054
0.8	1.2730	1.6860	1.8133	0.5831
1.0	1.1130	1.8325	1.8113	0.5534	1.4410	2.1790	1.9489	0.6892
1.2	0.9550	1.9755	1.7777	0.5230	1.2890	2.3560	1.9105	0.6492
1.4	0.7999	2.1120	1.7425	0.4926	1.1420	2.5850	1.8681	0.6069

(20°, 30°, and 40°) each associated with three values of N ($\frac{1}{2}$, $\frac{3}{4}$, and 1). The corresponding graphs for $\phi = 30^\circ$ and $\phi = 40^\circ$ are presented in Fig. 5. Supplementing information in Fig. 5, the design constants for $\phi = 20^\circ$, 30° , and 40° are given in Table 2 for the special case $C = 0$ and $C_s = 0$.

Design Example.—To demonstrate the use of a design chart such as Fig. 5, assume $H_s = 4$ ft, $H = 8$ ft, $H_w = 22$ ft, $W = 100$ lb per sq ft of pile, $\phi = 30^\circ$,

^{9a} A sheet of nine sets of Design Curves will be mailed upon the receipt of \$1.00 and your request for Proceedings-Separate No. C-165.

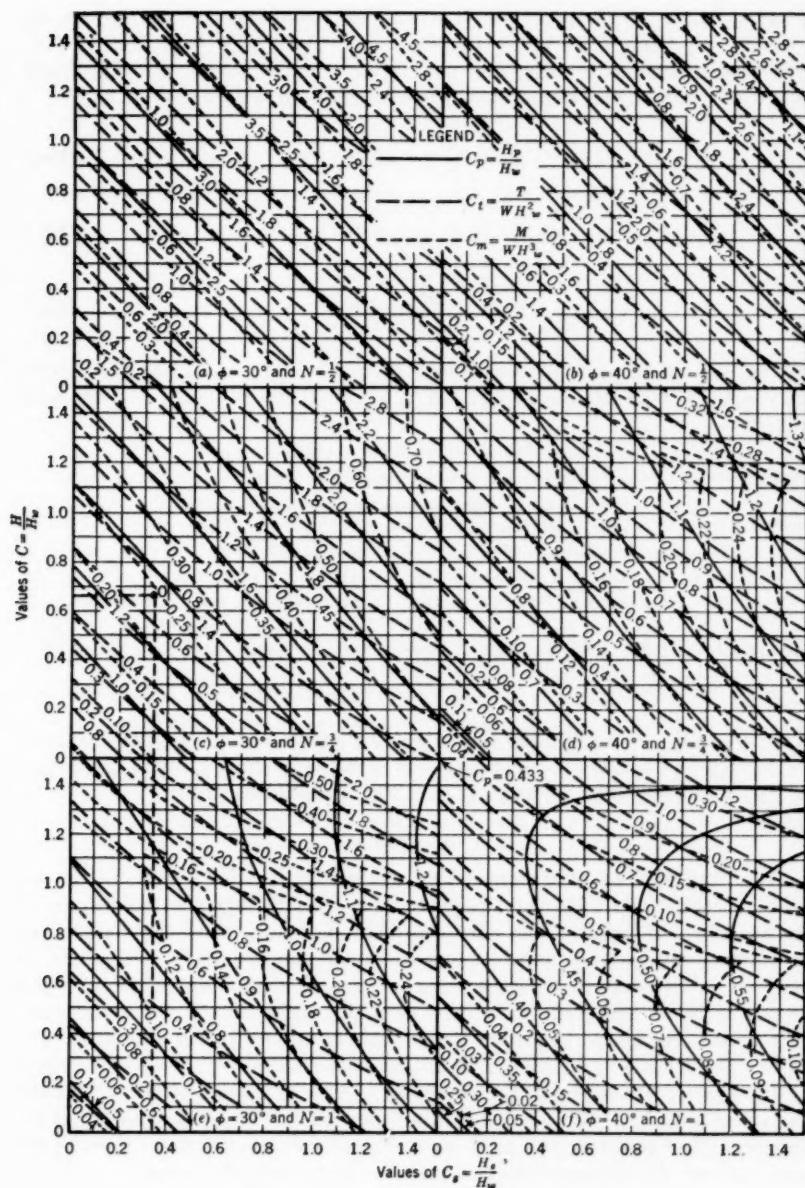


FIG. 5.—DESIGN CURVES FOR ANCHORED STEEL SHEET PILING

and $\phi' = 22.5^\circ$. With these data, it is possible to compute $C = H/H_w = 8/12 = 0.667$; $C_s = H_s/H_w = 4/12 = 0.333$; and $N = \phi'/\phi = 22.5/30 = 3/4$. With these values, enter Fig. 5(c) and locate point O where $C = 0.667$ and $C_s = 0.333$. Referring to the legend to identify appropriate curves, read

$$C_p = \frac{H_p}{H_w} = 1.34 \dots \dots \dots (39a)$$

$$C_t = \frac{T}{W H_w^2} = 0.71 \dots \dots \dots (39b)$$

and

$$C_m = \frac{M}{W H_w^2} = 0.24 \dots \dots \dots (39c)$$

Solving Eqs. 39 for appropriate values— $H_p = C_p \times H_w = 1.34 \times 12 = 16.06$ ft; $T = C_t \times W \times H_w^2 = 0.71 \times 100 \times 12^2 = 10,200$ lb per ft of pile; and

TABLE 2.—DESIGN CONSTANTS

Constant	Equation	ANGLE OF FRICTION, ϕ (For $C = C_s = 0$)								
		20°			30°			40°		
		0.5	0.75	1	0.5	0.75	1	0.5	0.75	1
C_p	12	1.850	1.125	0.770	1.130	0.660	0.410	0.762	0.408	0.225
C_t	17a	0.290	0.160	0.110	0.158	0.080	0.050	0.108	0.057	0.030
C_m	32	0.160	0.100	0.057	0.110	0.045	0.026	0.057	0.026	0.013

$M = C_m \times H_w^3 = 0.25 \times 100 \times 12^3 = 41,610$ ft-lb per ft of pile. To find the correct coefficients for values of ϕ and N between those given in the design curves, an interpolation curve similar to Fig. 4 should be drawn.

CONCLUSION

The development of dimensionless coefficients for the design factors required in sheet pile bulkhead design reduces considerably the tediousness of the analytical solution. The curves presented should prove very useful for preliminary design work and in many cases will suffice for the final design as well. It is not anticipated that the condition of maximum moment at the tie rod or below the mud line will be normally encountered—but these cases have been investigated to extend the possible usefulness of the charts.

Several designs have been made to compare results with those determined by the charts. In all cases the charts have yielded values within 2% of the analytical solution. Within the limits imposed by the assumptions of the design method selected, the method of dimensionless coefficients permits the most efficient development of charts for the steel sheet pile design.

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